# CHAPTER 3

# DESIGN STUDIES

3-1. General. The design of small boat harbor projects requires an understanding of the problem, assembly and evaluation of all pertinent facts, and development of a rational plan. The design engineer is responsible for developing the design rationale and sufficient alternative plans so that the economically optimum plan is evident and the recommended plan is substantiated. Applicable Corps of Engineers guidance is considered in the design. Pertinent textbooks, research reports, or expertise from other agencies may be used as source information. The usual necessary steps leading to a sound plan are outlined below:

a. Review appropriate ER's, EM's, ETL's and other published information.

b. Assemble and analyze pertinent factors and environmental data.

c. Conduct baseline surveys.

d. Select a rational set of design conditions.

e. Develop several alternative layouts with annual costs.

f. Select an economically optimum plan.

g. Assess environmental and other impacts.

h. Develop recommended plan.

l. Develop operation and maintenance plan.

3-2. Typical Engineering Studies. The following studies are normally considered for small boat harbor project design.

a. Water levels and datums

b. Waves

c. Currents

d. Shoreline changes

e. Sediment budget and channel shoaling

f. Design vessel or vessels

g. Baseline surveys

h. Design life, degree of protection, and design conditions

i. Channel width

j. Channel depth

k. Channel alignment

l. Turning basin

m. Basin and breakwater layout

n. Breakwater design

o. Dredging and disposal

p. Environmental impact

q. Model studies

r. Operation and maintenance

3-3. Water Levels and Datums.

a. General. Both maximum and minimum water levels and frequency, duration, and amplitudes of water-level fluctuations are needed for design of small boat harbor projects. Water levels can be affected by storm surges, seiches, river discharges, natural lake fluctuations, reservoir storage limits, and ocean tides. High water levels are used for prediction of wave penetration and breakwater heights. Low water levels are used to determine channel and moorage area water depth and breakwater toe design.

b. Tide Predictions. The National Ocean Survey (NOS) publishes tide height predictions and tide ranges. Figure 3-1 shows spring tide ranges for the continental United States. Published tide predictions are sufficient for most project designs; however, prototype observations usually will be required for verification of physical or numerical hydraulic models when used.

c. Datum Planes. Small boat harbor project features will be referred to appropriate low-water datum planes. The relationship of the low-water datum to the National Geodetic Vertical Datum (NGVD) will be needed for vertical control of construction. The low-water datum for the Atlantic and Gulf Coast is presently being converted to mean lower low water (mllw). Until the conversion is complete, the use of mean low water (mlw) for the Atlantic and Gulf

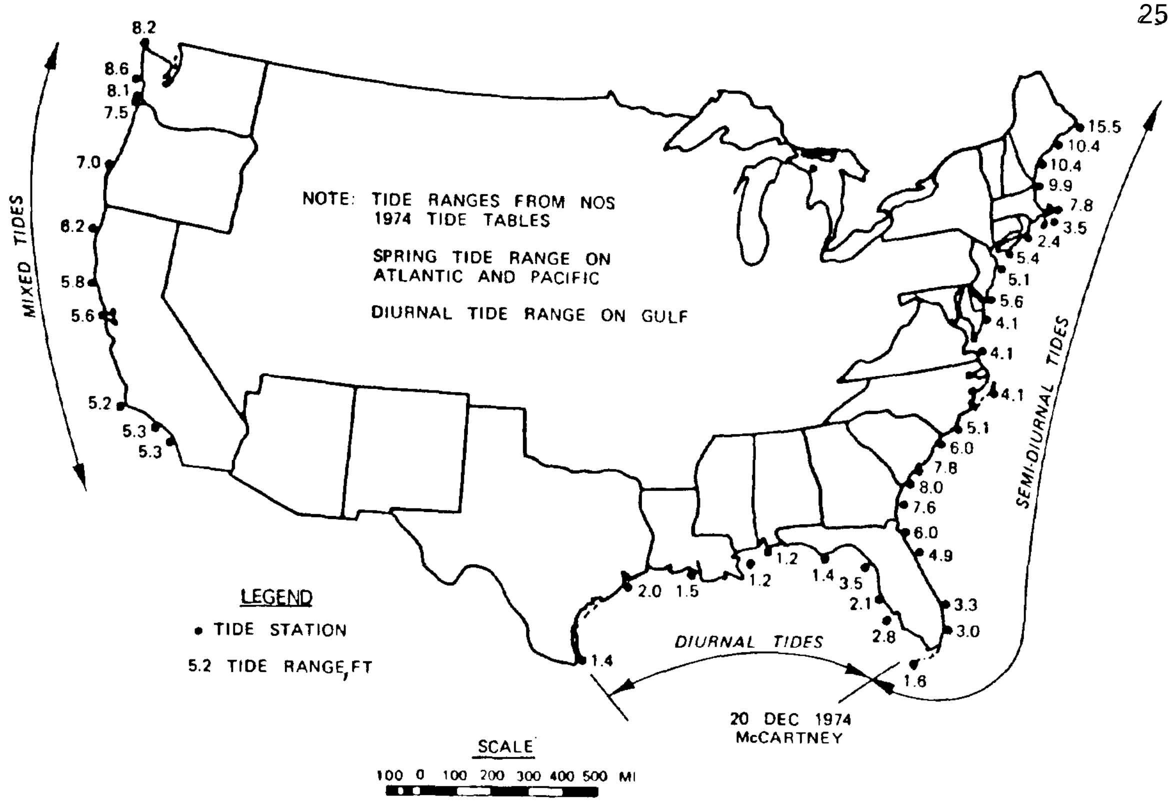


Figure 3-1. Ocean tide ranges.

Coast Low Water Datum (GCLWD) is acceptable. Other low-water datums are:

| | |
|---|---|
| Pacific Coast | - Mean lower low water (mllw) |
| Great Lakes | - International Great Lakes Datum (IGLD) |
| Rivers | - River, Low Water Datum Planes (Local) |
| Reservoirs | - Recreation Pool Levels |

3-4. Waves.

a. General. Naturally occurring wind waves and vessel generated waves require analysis and prediction. Wave conditions are needed for various elements of the project design. This allows reduction of channel dimensions where wave effects on vessel maneuverability diminish.

b. Wind Waves. Prediction of wind wave heights and periods can he made using techniques presented in the Shore Protection Manual (referenced), or the report titled "Determining Sheltered Water Wave Characteristics" (Vincent and Lockhart, 1983) Another source is wave hindcast information published by the Waterways Experiment Station (Resio and Vincent, January 1976, March 1976, November 1976, 1977, 1978) (Corson and Resio, 1981) (Corson, et al., 1981; Corson, et al., 1982) (Corson, Resio, and Vincent 1980). These hindcast wave heights and periods are applicable for deep water and require refraction and diffraction analysis to develop wave characteristics at the project site. The Shore Protection Manual (reference d) presents a method for calculating refraction and diffraction effects. If feasible, installation of wind and wave gages at the project site is strongly recommended. One year of wind and wave records is considered a minimum to verify or adjust wave predictions before the design is finalized.

c. Vessel Generated Waves. Passing vessels may generate larger waves than the wind. This is particularly true for boat harbors or ramps on rivers where passing deep draft vessels or barges may generate damaging waves. The height of waves generated by a moving vessel is dependent on the following:

(1) Vessel speed

(2) Vessel draft and hull shape

(3) Water depth

(4) Blockage ratio of ship to channel cross section

The effects of waves will depend on the height of the wave generated and the distance between the ship and the project site. An estimate of the height of a ship-generated wave can be obtained by assuming the wave height (crest to trough) will be equal to twice the amount of vessel squat. The wave height at the shore is then computed using refraction and diffraction techniques (reference d). The wave length would be equal to approximately one third of the vessel length. The method used to predict vessel squat is presented in paragraph 3-12. If vessel generated waves are considered the design wave, model tests or prototype measurements will be needed to verify or adjust the predictions. Additional information on the possible impact of vessel wakes may be obtained from (Camfield, Ray, and Eckert, 1980).

d. Selection of Test Waves from Prototype Data. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions can be based are usually unavailable for various project areas. However, statistical or deepwater wave hindcast data representative of these areas are normally obtained. Wave data used for various study sites along the Atlantic, Gulf, and Pacific Coast frequently can be obtained from the following:

(1) National Marine Consultants (1960)

(2) Surface Marine Observations (National Climate Center, 1976)

(3) Fleet Numerical Weather Center (1977)

(4) Meteorology International (1977)

(5) Saville (November 1954)

(6) Marine Advisors (1961)

(7) Synoptic Meteorological Observations (1971)

(8) Marine Advisors, Inc. (1964)

(9) U. S. Navy Hydrographic Office (1950)

(10) Bretschneider (1970)

(11) Corson, et al., (January 1981)

Wave data commonly used for study sites on the Great Lakes can be obtained from the following:

(1) Resio and Vincent (January 1976, March 1976, November 1976, 1977, 1978)

(2) Saville (1953)

(3) Sverdrup and Monk (1947)

(4) Arthur (1948)

(5) Bretschneider (1970)

(6) Cole and Hilfiker (1970)

3-5. Currents.

a. General. Currents can be tidal, river, or seiche induced. The currents can have a beneficial effect by promoting boat basin flushing. However, if the currents are too strong, then they can adversely affect vessel maneuverability in the channels and turning basins and cause problems with moored or anchored vessels. Current forces are also required for floating breakwater mooring system design. Prediction of current strength and duration is needed for selection of the design conditions. Prototype measurements are usually needed before the final design is complete.

b. Tidal Currents. Tidal currents for most coastal areas are published by the NOS. This information is sufficient for preliminary design. However, prototype measurements are needed for final design.

c. River Currents. River currents can be estimated by backwater computations of various flood discharges and verified by prototype measurement. Figure 3-2 depicts damage caused by floods and Figure 3-3 shows a method to separate river currents from a boat basin.

d. Seiche Currents. Large bodies of water like the Great Lakes can have seiches which produce currents in inlets or harbors with constricted entrances. These currents at nine harbors on the Great Lakes are discussed in Seelig and Sorensen (1977).

## 3-6. Shoreline Changes.

a. General. The natural growth or recession of the shoreline and offshore hydrography are needed to predict the impact of a project. If the project creates adverse impacts such as accretion or erosion, suitable mitigation measures such as sand bypassing or beach protection structures may be required.

Figure 3-2. River flood damage at Ventura Marina, California.

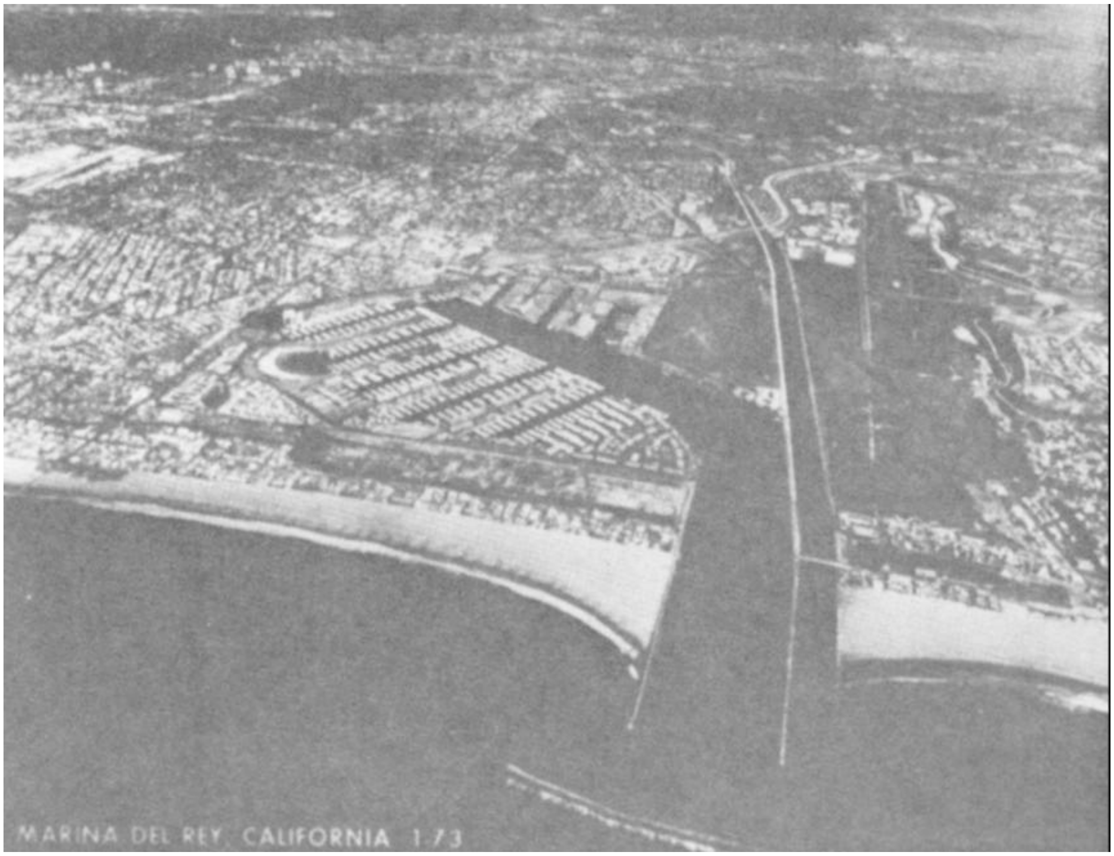


Figure 3-3. Separation of river flow from boat basin at Marina Del Ray, Los Angeles County, California.

b. Evaluation Methods. Historic changes can be obtained from old charts or photographs. The NOS survey sheets are a good source of information since they show actual soundings of most coastal areas dating bark to the early 1800's. Care must be taken when comparing old survey data to assure horizontal and vertical control are corrected to a common reference. Old photographs can give approximate indications of changes; however, quantitative comparisons are difficult because water levels (tide, lake fluctuations, or river stages) are usually unknown.

3-7. Sediment Budget and Channel Shoaling.

a. General. A sediment budget and channel shoaling estimate is needed to estimate maintenance dredging volumes and costs. The sediment budget will also indicate potential beach erosion areas.

b. Coastal Sediment Budget. Coastal sediment is moved primarily by waves. Therefore, a wave climate assessment and beach composition are required. The budget will identify sediment sources, volumes moved, reversals, and sinks (shoaling areas). The coastal sediment budget analysis method is described in reference d.

c. River Sediment Budget. A river sediment budget is similar to the coastal budget except the transport mechanism is river current and there are no reversals. The budget will identify sediment sources, volumes moved, and sinks (shoaling areas).

d. Channel Shoaling. The sediment budget will indicate approximate volumes of channel and mooring area shoaling. Movable bed physical models or mathematical models may be needed to refine shoaling estimates.

3-8. Design Vessel or Vessels. The design vessel or vessels are selected from comprehensive studies of the various types and sizes of vessels expected to use the project during its design life. There may be different design vessels for various project features. For example, sail boats may have the deepest draft for channel depth design, whereas fishing boats may have the widest beam for channel width design. The design vessel or vessels are identified as to various parameters affecting their maneuverability. There is considerable variation in the length, beam, and draft relationships of small craft. The following sources will help identify typical vessel dimensions:

a. "Boating Statistics," report of accidents, numbering, and related activities, published twice annually by the U. S. Coast Guard, 1300 E. Street N.W., Washington, D. C. 20591

b. "Boat and Motor Dealer," published monthly by Dietmeier-Van Zevern Publications, 344 Linden Ave., Wilmette, IL 60091

"Boat Builder." published twice annually by Davis Publications, Inc., 229 Park Avenue South, New York, NY 10003

d. "Boating Industry," published monthly by Whitney Communications, 850 3rd Ave., New York, NY 10022

3-9. Baseline Surveys. Physical and environmental surveys of the project. site are needed during the preconstruction design phase. Hydrographic and hydraulic survey data are also to be used for model construction and verification. The following surveys are usually needed:

a. Hydrographic

b. Beach profile

c. Waves: height, period, direction and duration (spectral distribution of wave energy may be needed)

d. Currents: velocity, direction, and duration

e. Sediment: suspended and bedload

f. Beach composition

g. Foundation conditions

h. Wind: speed, direction, and duration

i. Ice: frequency, duration, and thickness

j. Biological population: type, density distribution, and migration

k. Water quality

Dredge material water disposal sites will usually need a, d, j, and k baseline surveys.

3-10. Design Life, Level of Protection, and Design Conditions. The project design life and design level of protection are required before the design conditions can be selected. The economic design life of most small boat projects is 50 years. Level of protection during the 50-year period is usually selected by an optimization process of frequency of damages when wave heights exceed the design wave and the cost of protection. The elements that are to be considered in an economic optimization or life cycle analysis are

a. Project economic life

b. Construction cost for various design levels

c. Maintenance and repair cost for various design levels

d. Replacement cost for various design levels

e. Benefits for various design levels

f. Probability for exceedance for various design levels

The design level for a small boat harbor is usually related to wave heights and water levels. The severity of these events has a statistical distribution that can be ordered into a probability of exceedance. The exceedance probability is plotted against the design level (Figure 3-4). A series of project

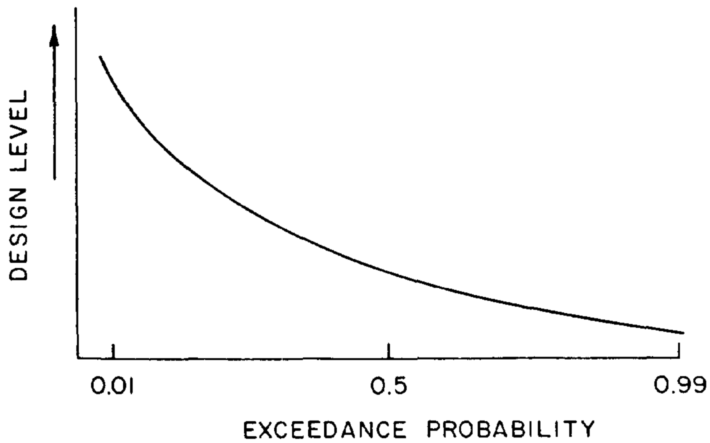


Figure 3-4. Exceedance probability versus design level

designs and cost estimates are developed for various design levels (wave heights). Construction cost is than converted to annual cost. Maintenance costs can be estimated by multiplying the exceedance probability of the design level by the construction first cost. The maintenance and repair cost should be compared with maintenance and repair cost for similar existing projects to assure realistic values. Some designs may call for partial or total replacement of a project feature one or more times during the project economic life. Average annual replacement costs are obtained by estimating the replacement years, determining replacement cost, and converting to present worth. The present worth value of the replacement cost is then converted to average annual cost by using appropriate interest rates and economic project life. The project cost curves usually look like Figure 3-5. Project benefits are compared with project cost to determine the economic optimum design level. Figure 3-6 shows this benefit cost comparison.

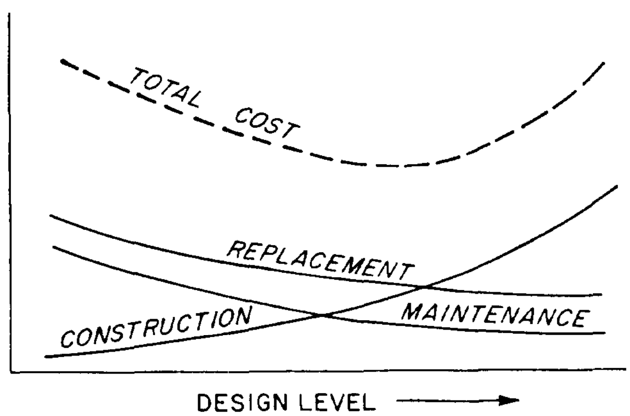


Figure 3-5. Project cost curves

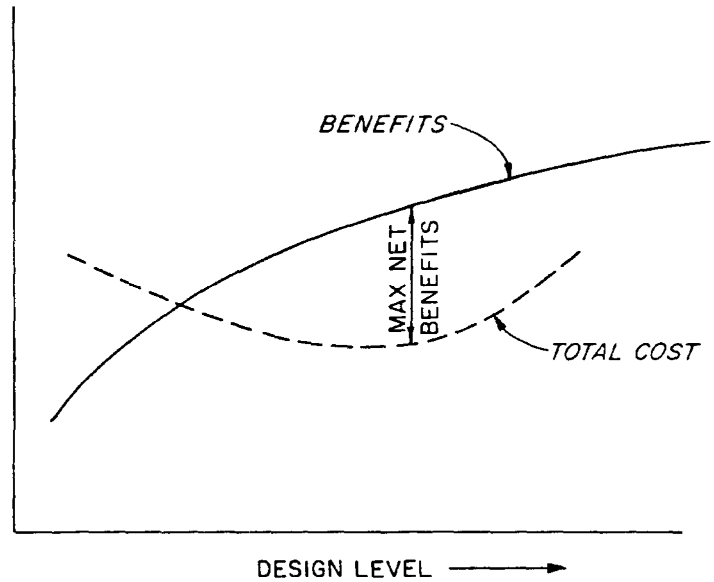


Figure 3-6. Benefits and cost versus design level

A separate analysis (Figures 3-5 and 3-6) will be needed for each alternative structure or project layout. Normally, the design level associated with the maximum net benefits will be selected for project design. Exceptions could be for harbors of refuge where a minimum design level is established or because of environmental concerns. If the net benefit point is not well defined, it may be prudent to select a higher design level.

3-11. Channel Width. A rational design is needed to allow safe and efficient passage of the vessels expected to use the project. Factors to be considered are:

a. Vessel size

b. Vessel maneuverability

c. Traffic congestion

d. Effects of wind, waves, and currents

Table 3-1 lists the recommended channel width elements as a percent of vessel beam for various degrees of vessel controllability (vessel steerage capability).

TABLE 3-1

Minimum Channel Element Widths (Committee on Tidal Hydraulics, 1965)

| Location | Minimum Channel Widths Needed in Percent of Beam — Vessel Controllability: Very Good | Good | Poor |
|---|---|---|---|
| Maneuvering Lane, Straight Channel | 160 | 180 | 200 |
| Bend, 26-degree Turn | 325 | 370 | 415 |
| Bend, 40-degree Turn | 385 | 440 | 490 |
| Vessel Clearance | 80 | 80 | 80 |
| Bank Clearance | 60 | 60 plus | 60 plus |

These widths can be increased for adverse wind, wave and current conditions, or for high traffic volumes (congestion). An example of a congested entrance would be a large recreation marina where most of the boats leave on a week-end morning and return in the evening (See Figure 3-7).

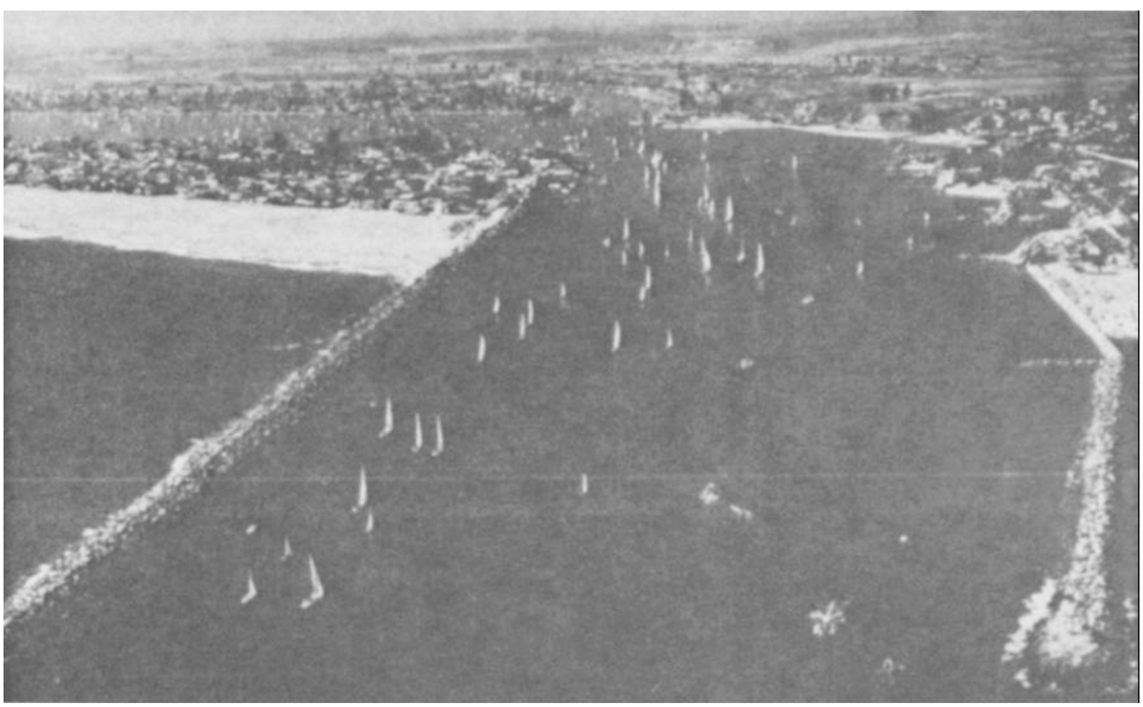

Figure 3-7. Entrance to Newport Bay, California

Interior channels generally need less width than entrance channels because wind, waves, and currents are less severe due to sheltered conditions. Widening on bends is usually required to allow safe turns. Physical hydraulic models with radio controlled model vessels or mathematical vessel simulator models can be used to evaluate the adequacy of channel widths.

3-12. Channel Depth.

a. General. Channel depths should be adequate for vessel draft and squat, wave conditions, and safety clearances. Additional depth is allowed in construction due to dredging inaccuracies. Overdepth dredging may also be included as an advance maintenance procedure. Vessel sinkage in fresh-water may also be a depth consideration. This sinkage is due to the density difference between fresh and salt water. The less dense fresh water will allow the boat to sink to a greater draft. Channel depths are usually measured from a suitable low-water datum. An extreme low-water level, such as a minus tide, may be used to increase the design channel depth when economically justified. Interior channel depths normally are not as deep as entrance channels because the wave action adjustment is normally less.. The type of dredge or other excavation equipment must be indicated to assure that it can operate in the selected channel depths. Tidal channel dimensions must be evaluated for stability to assure that rapid shoaling or erosion will not occur. Entrance channel and interior channel depth considerations are shown in Figures 3-8 and 3-9.

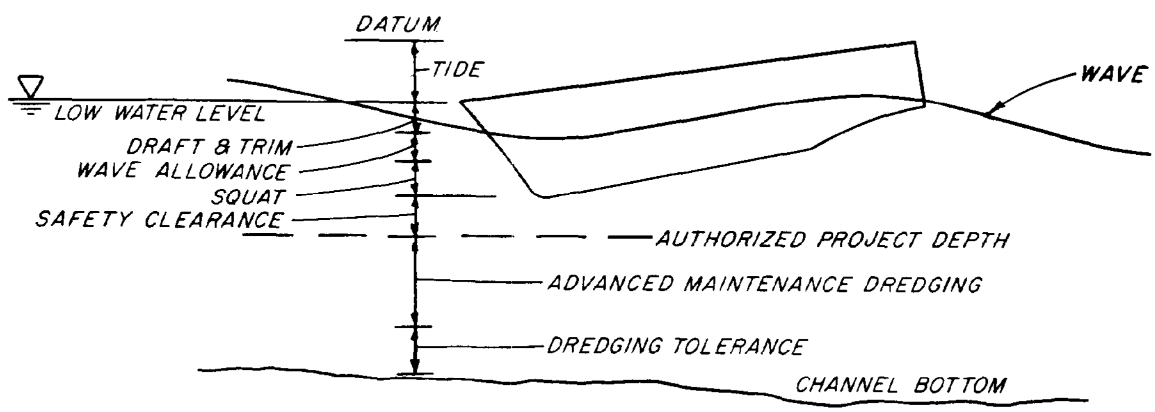


Figure 3-8. Entrance channel with wave effects.

b. Squat. Squat for small recreation craft moving at reasonable speed in entrance channels is generally taken to be one foot. Squat at low speed in interior channels, moorage areas, and turning basins is about 0.5 foot. Squat for large displacement hulls, such as fishing boats or ferries, is to be calculated. A ship in motion will cause a lowering of the water surface because of the change in velocity about the vessel, causing it to be lowered with respect to the bottom. Although this phenomenon also affects the ship's trim, the effect is minor and normally is neglected. The amount of lowering referred to

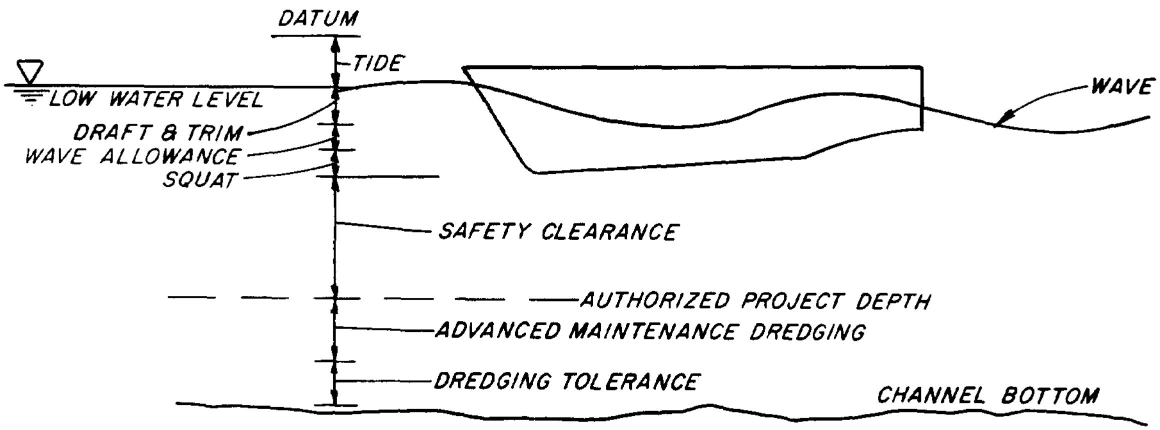


Figure 3-9. Interior protected channel.

as "squat" will depend on several factors, including the speed of the vessel, characteristics of the channel and vessel, and interaction with another vessel. The amount of additional channel depth to be provided for squat can be approximated using the following steps:

(1) Determine blockage ratio(s) of vessel submerged cross section to channel cross section from $s = A_s/WH_c$ where $A_s$ is vessel submerged cross section in square feet, W is average width of channel in feet, and $H_c$ is channel water depth in feet. A semiconfined channel (i.e., one in which the top of the dredged channel side slope is under water) is assumed to have the same cross section as a confined channel. This assumption will produce conservative results.

(2) Determine Froude number (F) from $F = \frac{V_s}{\sqrt{gH_c}}$ , where $V_s$ is vessel speed in feet per second, g is acceleration due to gravity (32.2 ft/sec$^2$).

(3) Apply calculated values of s and F to Figure 3-10 to obtain d , a dimensionless squat.

(4) Using the d value obtained from Figure 3-10, compute squat (Z) in feet from $d = Z/H_c$ or $Z = dH_c$ , where $H_c$ is depth of channel water. Squat will be greater when vessels are passing because the total blockage ratio is larger and must be considered in the design of channels for two-way traffic. In unrestricted waterways and open seas, squat is much less than in confined waterways because the submerged cross section of the vessel becomes a very small percentage of the waterway cross section.

c. <u>Wave Conditions.</u> Channel depth increase for wave action (wave allowance) is generally one-half the design wave height for small recreational craft. Pitch, roll, and heave must be evaluated also for larger vessels that use the

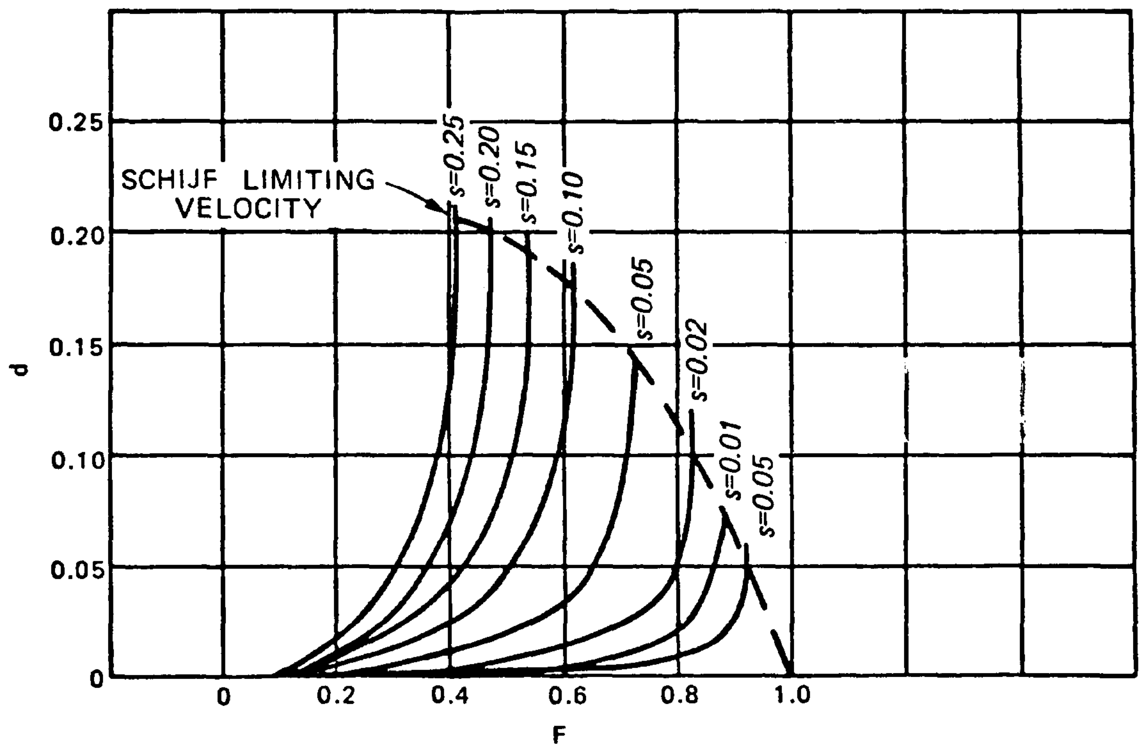


Figure 3-10. Dimensionless squat as a function of the Froude number. (From Committee on Tidal Hydraulics, Report 3).

channel. Vessel motions can be determined by prototype observations, physical models, or vessel-simulator mathematical models. The larger of the two wave allowances is to be used.

d. Safety Clearance. In the interest of safety, a clearance minimum of 2 ft is needed for channels with soft bottoms, such as sand or silt. When the channel bottom is hard, like rock or coral, a three-foot minimum clearance is required. The additional one foot is to compensate for the greater damage expected for vessels which strike a hard channel bottom.

e. Dredging Tolerance. In consideration of the inherent mechanical inaccuracies of dredges working in the hostile environment of adverse currents, fluctuating water surfaces, and non-homogeneous material, an additional segment of the channel cross-section referred to as dredging tolerance, is recognized. Dredging tolerance is not taken into account in theoretical channel design where a neat line is assumed; however, contract specifications must take it into account. Site conditions and presumed construction equipment should all be considered in assigning a value. Usually the value ranges from one to three feet, and the amount actually dredged in the tolerance zone is paid for at the same rate as for other pay segments.

f. Advanced Maintenance. Channel maintenance usually consists of removing sediment deposits on the channel bed. In channels where shoaling is continuous, overdredging is a means of reducing the frequency of dredging and still providing reliable channel depth over longer periods. Advance maintenance consists of dredging deeper than the channel design depth to provide for the accumulation and storage of sediment. Justification for advanced maintenance is based on channel depth reliability and economy of less frequent dredging. Estimates of channel shoaling rates (discussed in paragraph 3-7) are used in the justification for advanced maintenance dredging. Several depths should be considered to optimize the advanced maintenance allowance, but it must be noted that deeper channels will tend to be more efficient sediment traps and could shoal more rapidly.

g. Sinkage in Fresh-water. Sinkage will be a channel depth factor for large design vessels (fishing boats or other commercial craft) which pass from seawater into freshwater. The submerged depth is increased by 3 percent in freshwater because the density of seawater is 1.026 (64 pounds per cubic foot) and fresh water is 0.999 (62.4 pounds per cubic foot). Most small boat projects can delete this consideration because of their vessel's sizes. For example, a design vessel with a draft of 6 feet in salt water will have a draft of 6.2 feet in fresh-water.

3-13. Channel Alignment.

a. Entrance Channels. Entrance channels normally follow the course of the deepest bottom contours. This alignment usually requires the least initial construction dredging, and currents often follow this path, which is desirable for navigation. An alternative alignment would be the shortest route to deep water. Layout of the entrance channel alignment should consider direction of predominant wind and waves and their effect on navigation. Channel alignments dredged through shoals or bars tend to shoal rapidly and generally should be avoided, if possible. Alignments should avoid the insides of river bends because of high shoaling rates. Breakwaters or jetties paralleling the channel may be required to maintain a desired alignment and their design may require a physical model investigation. Movable bed physical models can be used to estimate relative shoaling rates for various channel alignments. Fixed bed physical hydraulic models, with radio controlled vessels and imposed waves or vessel simulator models, can assess transit safety of alternative alignments. Alignments should minimize the number of bends. This will ease navigation and reduce the number of aids to navigation.

b. Access Channels. Interior channels generally provide access from the entrance channel to the turning basin and moorage area. Therefore, layout of these elements must be coordinated.

3-14. Turning Basin.

a. General. The turning basin is generally provided to allow vessels to change direction without having to back for long distances. The basins are usually located at the end of interior access channels and/or at boat ramps.

b. Turning Basin Size. The size of the basin will depend on the maneuverability of vessels using the basin. It should be large enough to allow turning of small recreational craft without backing, (vessel turning radius). This distance can be obtained from observation. Larger commercial vessels may be required to maneuver forward and reverse several times to turn if such traffic is infrequent. Turning basins at boat ramps may require additional space to allow waiting areas for several boats while the ramp is occupied.

c. Turning Basin Depth. Depths should be consistent with connecting channels and provide adequate allowances for squat, wave action, and safety clearance. Squat of about one-half foot is normally adequate.

3-15. Moorage or Anchorage Area.

a. Size. Moorage areas need sufficient area to allow berthing piers and interior channels to accommodate the intended fleet. Anchorage areas must safely accommodate the intended fleet considering vessel movement when at anchor. Maximum allowable wave heights generally are limited to one foot in berthing and two feet in anchorage areas.

b. Depth. Depth should accommodate draft, trim, wave action, low tide, and a minimum one-foot safety clearance.

3-16. Basin and Breakwater Layout.

a. General. The basin layout will include breakwaters, piers, turning basin, interior channels, boat ramps, anchorage areas, and other marine features. The layout must show that the anticipated fleet can be adequately accomodated. Appendix A presents an inventory and details of boat basin layouts which have been model tested.

b. Breakwater Layout. Breakwaters, if needed, will provide protection to interior channels, moorage areas, and other basin elements. Several breakwater layouts, and the associated costs, usually will be needed to indicate the optimum arrangement. Allowable wave heights may be different in various basin elements. For example, a two-foot wave may be acceptable in moorage areas for large fishing vessels, where a one-foot wave may be the maximum acceptable at a boat ramp. The acceptable wave heights will depend on the vessel sizes and types of moorage (piers or anchorage). Wave penetration studies are required to show expected wave conditions in all channel and basin areas. Waves inside the basin can result from refraction, diffraction, and breakwater overtopping and/or transmission. Model studies (physical or mathematical) can be used to determine optimum entrance configuration and wave heights inside the basin. Figure 3-71 shows a typical three-dimensional harbor model. Reference d presents analytical methods used to predict wave refraction and diffraction.

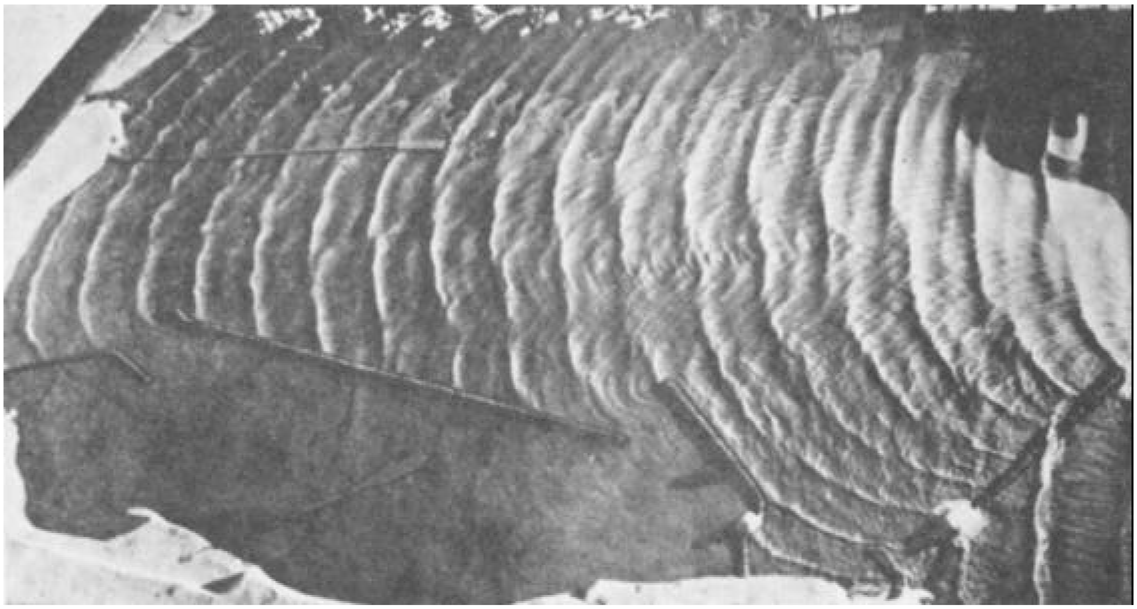

Figure 3-11. Model study for proposed harbor at Port San Luis, California. Note how proposed breakwaters attenuate waves from the South.

c. Long-Period Wave Amplification or Oscillation. An analysis of wave amplification or oscillation modes may be required on ocean coasts where long-period waves are prevalent. Certain geometric configurations may result in damaging wave conditions inside the basin and/or treacherous currents in the entrance channel. Procedures outlined in reference d can be used to evaluate amplification and harmonic oscillations. If an analysis determines this may be a problem, a physical model or a mathematical model investigation can be used to verify the problem and investigate solutions. A case history of surging in a small boat basin and the solution developed with model tests is presented in Weggel and Sorensen (1980).

d. Pier Layout. Guidance for minimum clearances for piers and interior channels is presented in references e and f. The detail necessary for the pier layout is shown in Figure 3-12.

3-17. Breakwater Design.

a. General. Breakwaters should be stable for all imposed design loads

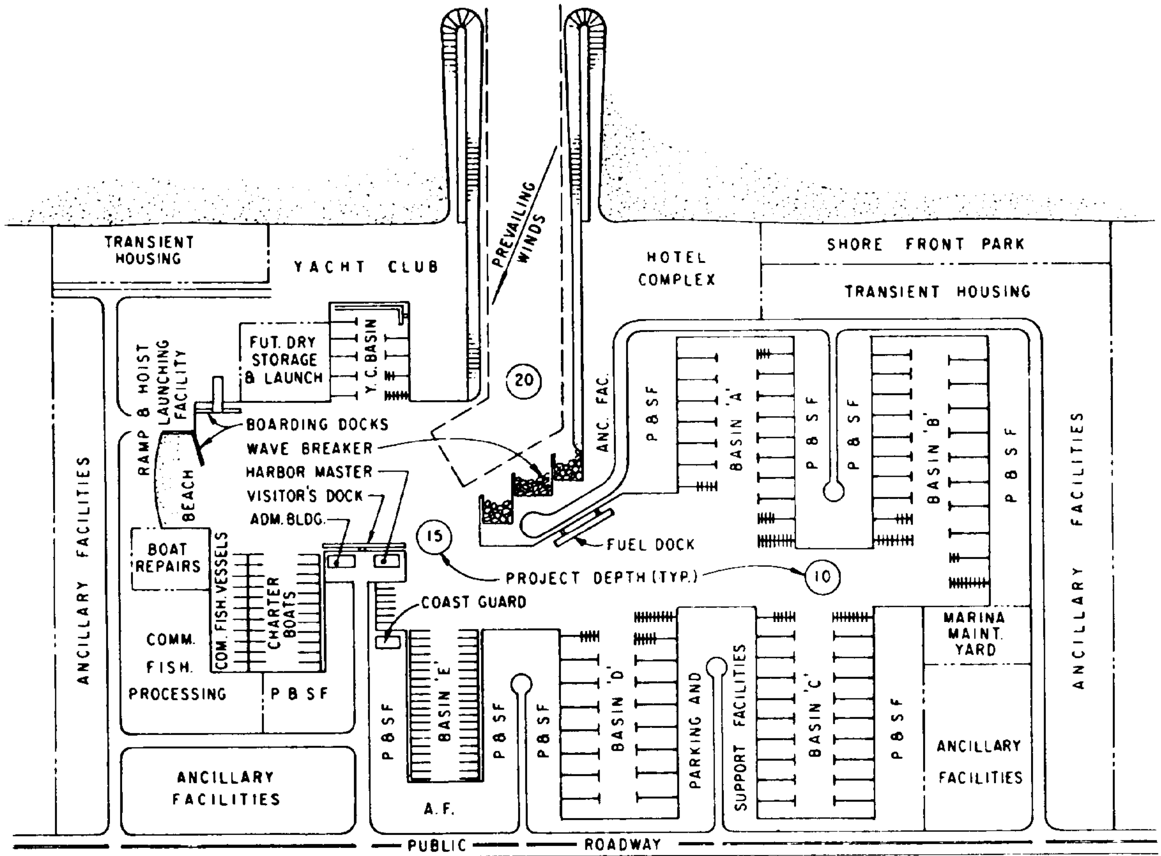


Figure 3-12. Schematic layout of a marina (for illustration only; not a recommended layout).

including waves, ice, and impact from debris and/or vessels. The design conditions are determined from the optimization process described in paragraph 3-10.

b. Types. Typical types of breakwaters used are rubble mound, timber pile, cellular sheet-pile, and floating structures (reference b and d). Bottom connected breakwaters can be designed to either prevent overtopping or allow some overtopping for harbor flushing. These breakwaters require firm bottom conditions to sustain their weight. Water depths are usually limited to 30 feet or less due to the high construction cost for these structures in deep water. Floating breakwaters can be used for sites with deep water, poor foundations, and/or where water circulation (i.e., improved water quality) is desirable. However, present designs are limited to design waves equal to or less than about 4 feet high with 4 seconds or less periods (Hales, 1981). Design procedures for various breakwater types are presented in references b and d. Generally two or three suitable breakwater types should be designed and cost estimates prepared to show the least cost alternative. For a valid comparison, estimates must include construction, maintenance, and replacement annual cost. The use of published stability coefficients are acceptable for preliminary design; however, final design will usually require two- and/or three-dimensional physical model tests constructed at a scale large enough to insure negligible scale effects.

3-18. Ice.

a. General. Ice may be the controlling factor for site selection, layout, and structural design of small-boat harbors in northern regions. Wherever ice can occur the following should be considered. Occasionally historical data will be sufficient, but usually water temperature, ice thickness, and tide and seiche effects on water levels must be measured. Ice can damage spring piles, finger piers and the other light construction in a small-boat harbor and this should be brought to the attention of the operator. In some areas ice may become so thick that continued use of the harbor is uneconomical and the harbor must close for a portion of the year. However, with proper design consideration, the length of this period of closure can be minimized. Physical modeling of some sites may be necessary to determine ice movement and accumulation patterns. Reference a provides information on ice forces and ice control measures.

b. Site Consideration. Open coast harbors built seaward from the shoreline and protected by massive breakwaters are seldom affected to any large extent by ice. Longshore currents or prevailing winds will cause ice transport, and the breakwater design should be such that this ice will not be trapped. If trapped, it should be easily flushed out by tides and currents. Breakwaters designed to withstand large waves are usually not damaged by ice with the exception of walls, railings, lights, or other structures on top of the breakwater that can be severely damaged when ice rides over the breakwater. Ice forces may be the controlling design load for breakwaters built in mild wave environments. Harbors built inland experience additional ice problems. Protection may be needed at moorings for very thin fresh water ice flowing downstream with each ebb tide. The incoming seawater may have a temperature as low as 29 deg F. This heat sink combined with very cold nights results in fresh water ice on the order of 1/2 inch thick which may damage hulls and mooring lines. Consideration must be given not only to the river ice which comes down during spring break-up but also those floes floated off the tidal flats during unusually high tides. Some sites such as Cattaraugus Creek (page A-50) have obstructions at the river mouth which trigger ice jamming and subsequent inland flooding. Even without a harbor-mouth bar, the ice may pile-up along the shoreline, sometimes called a windrow, and create the same effect. Construction of an offshore, detatched breakwater to force the windrow formations further seaward/lakeward and create two channel entrances has helped this problem. Where icebreaker services are available, the design should be coordinated with the provider to ensure that adequate depth and maneuvering room will exist for these specialized vessels.

c. Ice Forces. Lightly loaded piles can be jacked up when ice which is frozen to the pile is subject to vertical movement by tides and seiche as shown in Figure 3-13. The long period oscillations allow the sheet ice to


Figure 3-13. Damage to piles caused by the vertical movement of ice.

freeze at the pile and buoyancy forces acting on the entire sheet may lift the pile before the ice fails. Unfortunately, it takes a larger force to drive the pile so the second half of the oscillation does not return the pile to its original position. Figure 3-14 shows a typical pile driven narrow end down. A vertical fiberglass, PVC or plastic vertical sided sleeve, as shown on the right side of the figure, provides a surface along which the ice can slip. A more detailed discussion of the jackets and their performance is found in reference a. Figure 3-15 shows how a number of piles can protect each other when located on the order of less than 20 to 25 feet apart as long as the end piles are deep or well loaded. Here jackets may not be needed. Within a protected harbor, horizontal ice forces are not normally a problem. Thermal expansion of the ice cover is small and the structures are usually sufficiently stable.

d. Ice Control Methods. Bubblers, booms, air screens, warm effluent,

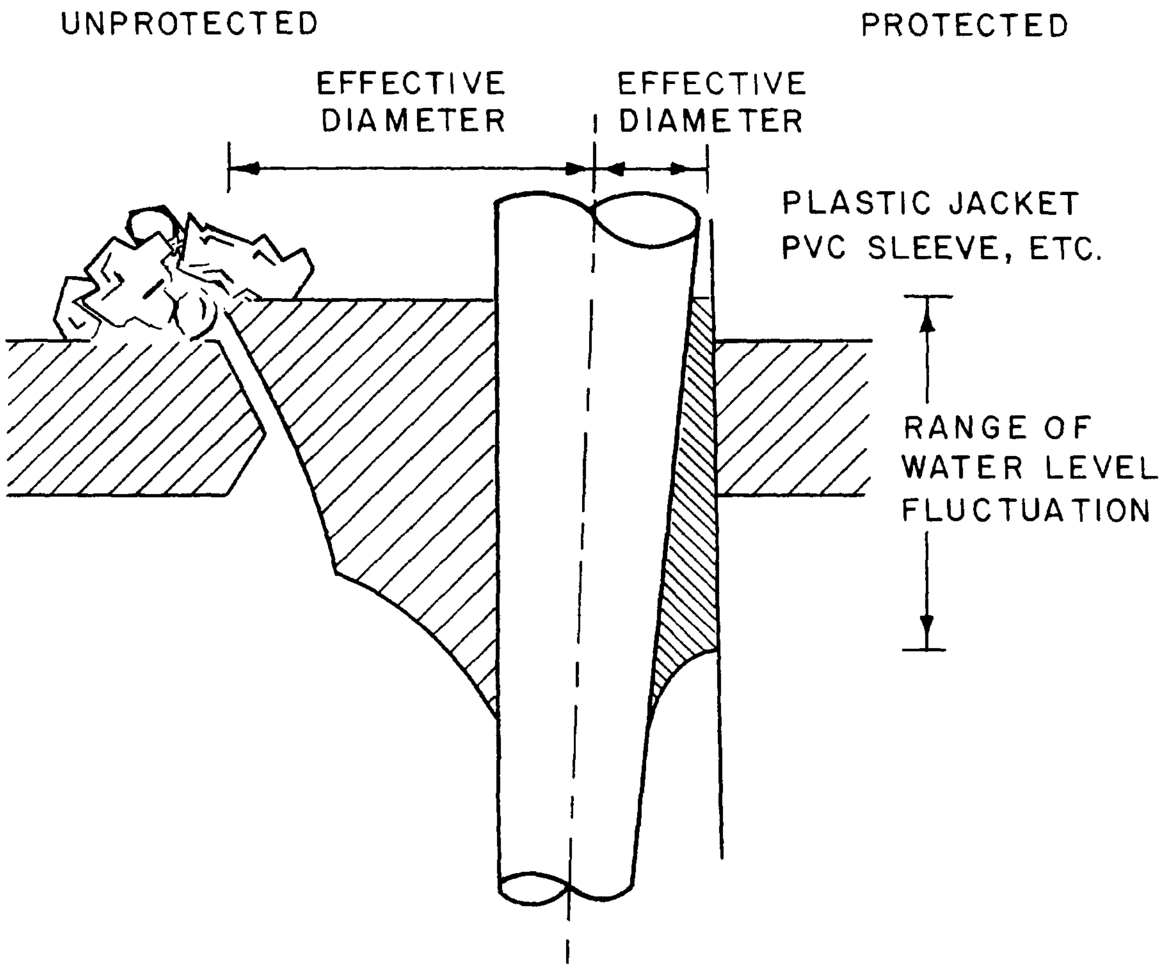


Figure 3-14. Typical pile driven (narrow end down) showing protection and nonprotection from ice.

piles, detached breakwaters, and artificial islands are all schemes to control ice. These methods are briefly described below.

(1) Bubblers. Bubblers discharge air at some depth, usually the harbor bottom. As the air rises to the surface, the bubblers entrain the warmer, water which has been trapped at the bottom during surface freezing. This warm water prevents additional ice formation or may keep the area above the bubbler ice free. Bubblers are used to protect docks, moored boats, and piles from ice action. Since they depend on warmer bottom water which is not always present, due to mixing in rivers or the presence of seawater, one must measure water temperatures before considering their use. It must be realized that only a finite amount of heat exists in the water and bottom sediments.

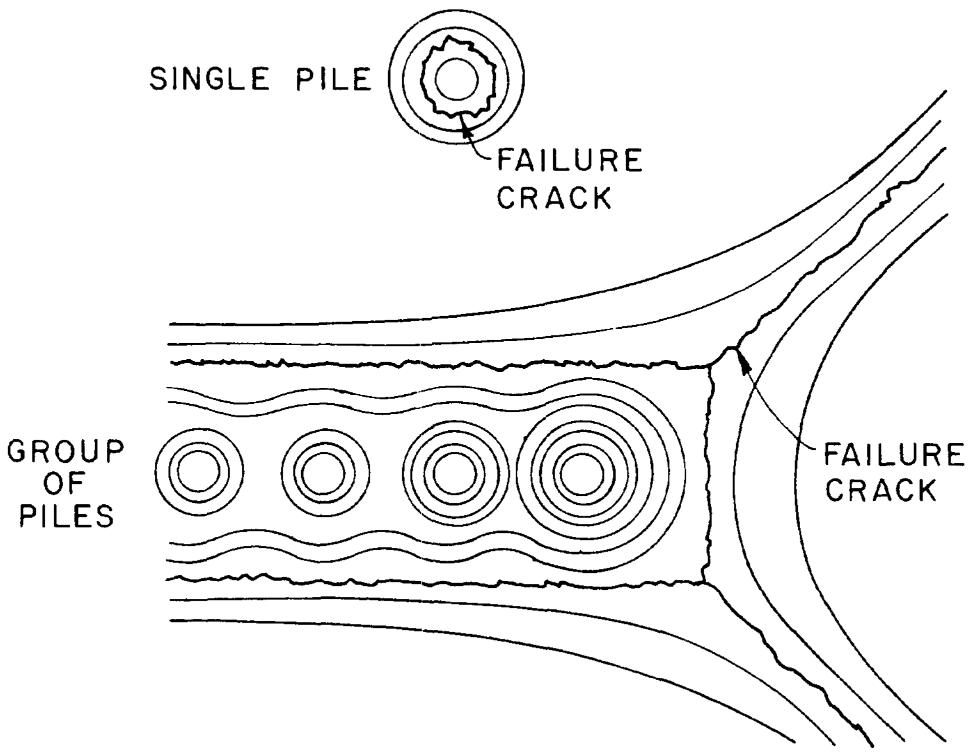


Figure 3-15. Contour lines of stress.

(2) Booms. Booms axe installed to retain moving ice, usually on rivers. Large timbers or pontoons are connected to wire ropes and anchored across a portion of, or the entire stream. During normal flow the ice floes will be retained and subsequently freeze together to form an intact ice cover. During a heavy ice run such as spring breakup, the ice will ride over and beneath these pontoons and so the boom is usually self-protecting. Booms are used primarily to form an ice cover in reaches where the ice cover needs more stability and support. A primary benefit is that the river water is insulated from rapid cooling and the growth of frazil ice is minimized. Ice booms are the primary means of ice control on rivers with winter navigation.

(3) Air Screens. When the need arises to divert, rather than retain, moving ice and at the same time permit vessel passage, an air screen works very well. Like a bubbler, air is discharged at some depth but in much larger volumes. The large volumes of water entrained, form an outward current upon reaching the surface. A line of air holes forms a line of diverging current on the surface across which ice passage is prevented under favorable conditions. Air screens also work well against debris but have not been successful in streams where the velocity exceeds 1.5 feet per second.

(4) Warm Water. Warm effluents, if available, are often thought to be the panacea for ice problems. It should be remembered that the effluent will

quickly mix with the colder water. Warm effluent can be used effectively with a bubbler. A point to remember, for those designing for cold climates, is that cold air on top of warmer open water makes fog. A thin layer of ice through which boats can move if often preferable to completely open water.

(5) Piles. Pile clusters, rock filled cribs, and tire or log booms can be used effectively as an ice control measure as shown in Figures 3-16 and 3-17.

(6) Detached Breakwaters and Artificial Islands. These features can be used to divert drift ice away from moored or anchored vessels.

3-19. Dredging and Disposal.

a. General. When dredging is required, a study is needed to determine the dredging, transport method, and the short and long-term disposal impacts. Beneficial uses of dredge material should be evaluated. Guidance on dredging, disposal, and beneficial uses can be obtained from reference c.

b. Dredges. Suitable types of dredge equipment should be specified to determine their capability of operating in the shallow project dimensions that are often specified for small boat projects. Pipeline dredges are normally used for soft materials; and blasting, with clam shell shovel removal, is used for rock or coral excavations.

c. Disposal Methods. Dredge material can be disposed of in open water or behind confinement dikes. These disposal areas can be in water areas or upland sites. Contaminated dredge material is generally disposed of behind containment dikes with careful monitoring of return water quality.

3-20. Sand Bypassing.

a. General. Sand bypassing should be considered when evaluating various harbor layouts and their potential impacts. Although sand bypassing has been used primarily at harbors on open coasts, its principles and many operational techniques apply to riverine harbors as well.

b. Types of Bypassing.

(1) Natural. One goal of the harbor design should be to maximize natural bypassing. Model investigations with tracer material or a movable bed can provide valuable information on the natural bypassing potentials of different harbor configurations. (Melton and Franco 1979) describe some general investigations of this type regarding riverine harbor designs.

(2) Artificial. Artificial bypassing, when used, is usually installed after the harbor has been in place long enough to determine the harbor's interactions with its surroundings. However, the possible need for artificial

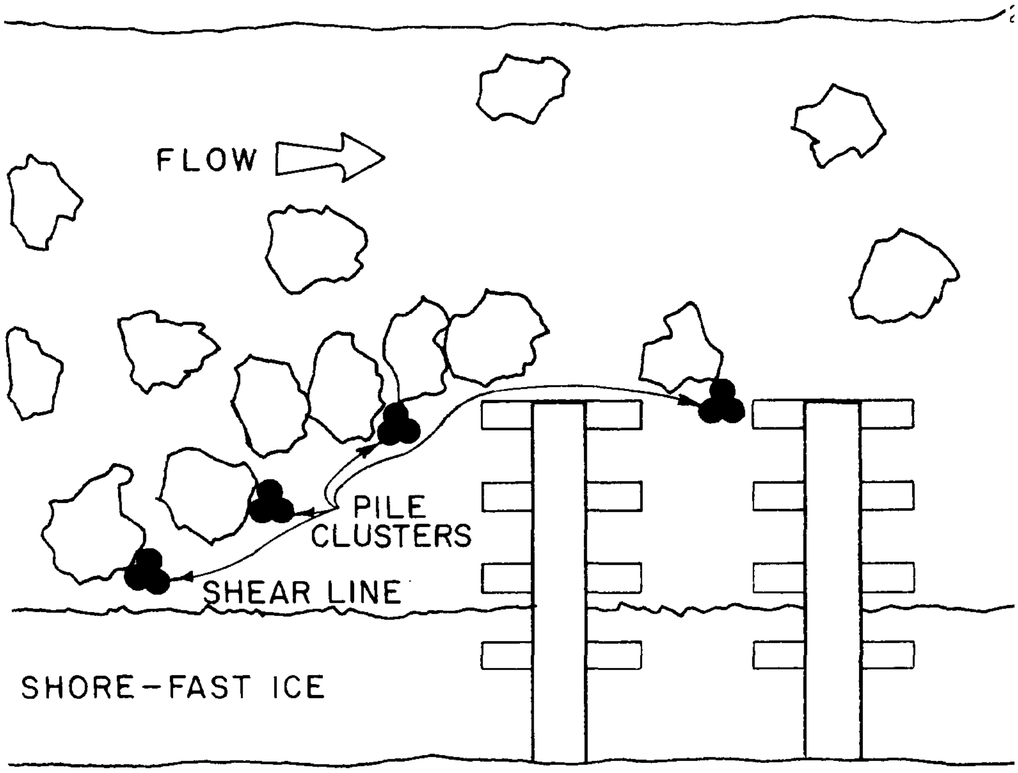


Figure 3-16. Controlling ice flows with pile clusters.

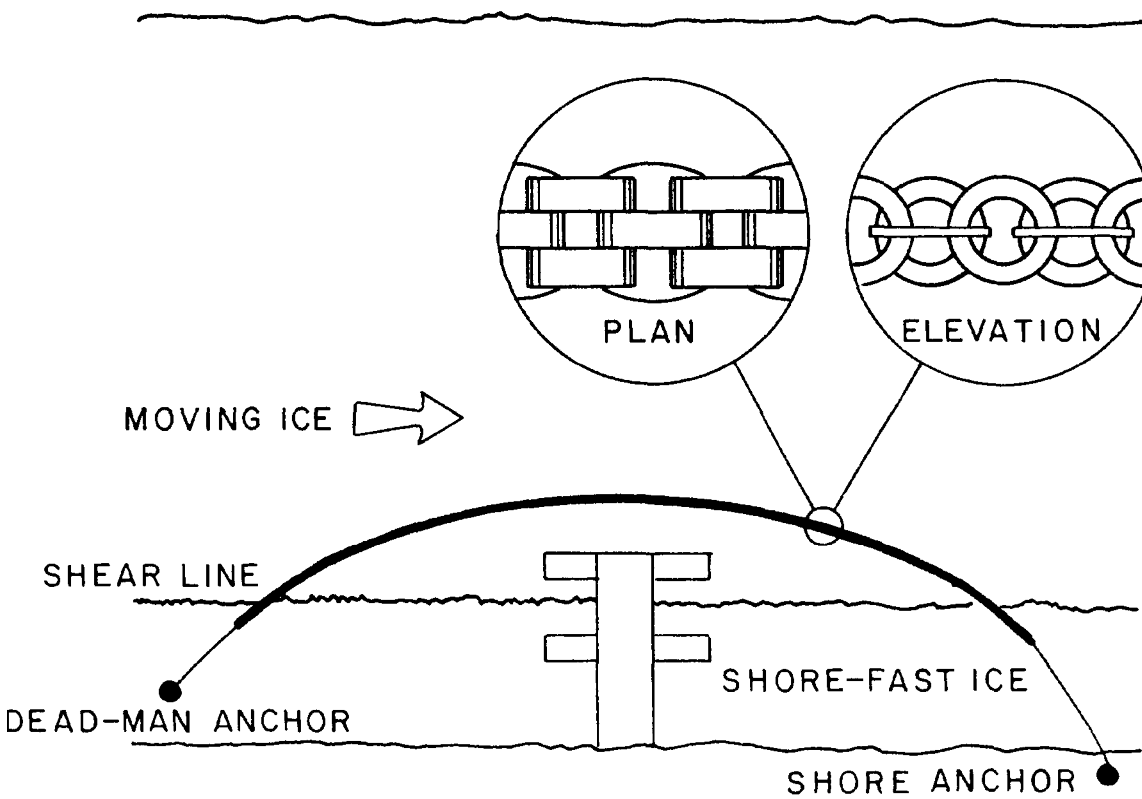


Figure 3-17. Controlling ice flows with a floating tire breakwater.

bypassing can be evaluated during the design process. Provisions for future artificial bypassing can be incorporated into the harbor layout.

c. Modes of Operation. Natural bypassing requires continuity of sediment transport to some degree. If the harbor layout produces a drastic or abrupt alteration in sediment flow patterns, natural bypassing may be restricted forever or impeded for some time while the adjacent shoreline or river bed adjusts. In general, harbor structures will reduce natural bypassing if they are placed perpendicular to sediment flow paths or if they extend into water too deep or too slow for normal sediment transport. Wide or deep navigation channels or detached breakwaters on open coasts also will limit natural bypassing. Artificial bypassing can be accomplished by intercepting moving sand or by removing deposited sand from a particular area. (Richardson and McNair 1981) discuss these modes of operation and other concepts involved in planning an artificial bypassing system. In small-boat harbor design, provisions can be made for artificial bypassing by creating zones where sediment movement will be channelized close to harbor structures or by designing for sediment deposition in particular locations. In coastal harbor design, weir sections in conjunction with jetties are sometimes used to trap sand within harbor structures for artificial bypassing (Weggel 1981). Detached breakwaters can perform a similar function.

d. Frequency. Both natural and artificial bypassing can be on a periodic or relatively continuous basis. Sediment movement may be blocked by a harbor under normal conditions, but natural periodic bypassing may occur during storms or times of high sediment transport rates. Artificial bypassing is usually associated with removing sand from a deposition area. Artificial bypassing may be periodic or continuous. At harbors where sediment transport is moderate and predictable, periodic artificial bypassing can be cost-effective. By providing deposition areas at several harbors in the same region, one bypassing system such as a dredge can be moved from harbor to harbor, performing periodic artificial bypassing at each.

e. Types of Artificial Bypassing Systems.

(1) Fixed. Bypassing system is fixed at one location in or adjacent to the harbor. This type system usually operates in an intercepting mode on a relatively continuous basis.

(2) Mobile. Entire system can be moved to different areas of the harbor or to other harbors. Such systems usually act on a periodic basis to remove deposited sand.

(3) Semimobile. System has mobility within a well-defined area of the harbor. This type system may operate in several combinations of modes and frequencies.

f. Equipment for Artificial Bypassing. Almost any item of equipment capable of excavating and/or transporting sediment might be used in an artificial bypassing system. Equipment commonly used includes:

(1) Hydraulic Dredges. Hydraulic pipeline dredges are probably the equipment most frequently used for artificial bypassing. They are almost always used for mobile periodic bypassing. They usually operate in deposition areas such as impoundment basins behind weirs and detached breakwaters and from navigation channels. They also can be used to mine sediment accumulations adjacent to harbors. (Savage 1957) gives a description of such an operation. Flexibility and high capacity are advantages of such equipment, while susceptibility to wave action, navigation interference, and potentially high mobilization and demobilization costs are some disadvantages. Trailing suction hopper dredges can also be used for bypassing shoal material from navigation channels. With a pumpout capability, they can move sand from their hoppers through a pipeline. Smaller split-hull hopper dredges can sometimes be used to dump dredged material in shallow water as a means of artificial bypassing (Sanderson 1976).

(2) Fixed Pumping Plants. The second most frequently used type of artificial bypassing equipment is the fixed pumping plant. In its simplest form, this plant consists of a solids-handling pump, a suction pipe extending into the water, and a discharge pipe to carry sediment past the harbor. Such plants are usually used to intercept moving sand and are operated relatively continuously. The plants are located most often on a harbor structure such as a jetty. A number of authors describe fixed pumping plants at various sites (Caldwell 1950, Senour and Bardes 1959, Rolland 1951, De Groot 1973, McDonald and Sturgeon 1956, U. S. Army Corps of Engineers 1956). Potential advantages of fixed pumping plants are low operating cost and dependability. Disadvantages include limited reach and lack of flexibility. Fixed pumping plants must be located and sized with extreme caution to insure that they receive adequate supplies of moving sediment but do not become "landlocked" by deposited sediment.

(3) Jet Pump Systems. Jet pump artificial bypassing systems were developed in the 1970's to fill the void between small fixed pumping plants and large hydraulic dredges. Such systems use one or more jet pumps (also called eductors) located on or below the bottom. The jet pumps are driven by centrifugal water pumps and operate by digging cone-shaped craters in the bottom. These craters act as deposition areas for moving sediment. A simple jet pump system usually includes a dredge pump to boost sand through the discharge pipe. One major advantage of a jet pump system is flexibility. The jet pumps can be either fixed or moved about in a wide variety of configurations to suit project requirements. The centrifugal pump, dredge pump, and other major plant items can be located on land, on harbor structures, or on floating platforms. The system is relatively resistant to wave action and can be configured to avoid navigation interference. (Richardson 1980) describes a trailer-mounted portable jet pump system designed to service several small harbors in the Great Lakes. Disadvantages of a jet pump artificial bypassing system

include power inefficiency, limited reach from shore, and susceptibility to debris plugging the jet pump suction. (Richardson and McNair 1981) provide detailed information for planning and sizing a jet pump artificial bypassing system.

g. Examples. The following example projects illustrate a wide range of characteristics, conditions, and equipment in sand bypassing.

(1) Santa Cruz, California. Figure 3-18 illustrates the layout of harbor structures at Santa Cruz, the nature of shoaling in the channel, and the type of artificial bypassing equipment used. Harbor shoaling occurs rapidly at Santa Cruz, usually in the winter. By late winter or early spring, the harbor entrance is completely closed and natural bypassing takes place. In the spring, a medium-size hydraulic dredge is placed in the harbor and begins removing the shoal, bypassing it to the downdrift beach. This operation takes approximately two months and by-passes 90-100,000 cubic yards of sand. By the time boat traffic demand becomes large, the harbor is clear and remains so for the summer and fall. Santa Cruz is an example of both natural and artificial bypassing on a periodic basis. Artificial bypassing is accomplished by mobile equipment (a hyraulic dredge) removing sand from a deposition area (the navigation channel).

(2) Marina di Carrara, Italy. Figure 3-19 shows the general layout of the Marina di Carrara harbor structures and artificial bypassing system. Sediment transport at Marina di Carrara is moderate, relatively regular, and mostly in one direction as shown. The harbor structures were built seaward from the shoreline and form an almost complete barrier to natural bypassing. The fixed bypassing system shown in Figure 3-19 was installed to move sand past the harbor to eroding beaches downdrift. It consists of a device similar to a dredge but mounted above the water surface on a circular concrete pier. This "rotating dredge" can pump sand up to four miles through a discharge pipe with four booster pump stations. Average pumping capacity is 130 cubic yards per hour, and the system operates relatively continuously. See DeGroot (1973) for more detail.

(3) Rudee Inlet, Virginia. Rudee Inlet has been the site for a fixed artificial bypassing system (McDonald and Sturgeon 1956), a mobile one (a hydraulic dredge), and a semi-mobile system. The semi-mobile system and present harbor structure layout are shown in Figure 3-20. The harbor incorporates a weir section in one of the jetties and a deposition area immediately behind the weir. Sediment transport is mostly in the direction shown at the rate of 70-120,000 cubic yards per year. A large portion of the total sediment load moves over the weir into the harbor, but some natural bypassing probably occurs along a bar seaward of the harbor entrance. The semi-mobile artificial bypassing system was installed as an experiment in 1975 and left there to be operated by local authorities. It consisted of two jet pump modules (Richardson and McNair 1981) which could swing in large arcs to remove sand from the deposition area, a pump house on shore, and a discharge pipe carrying sand to downdrift

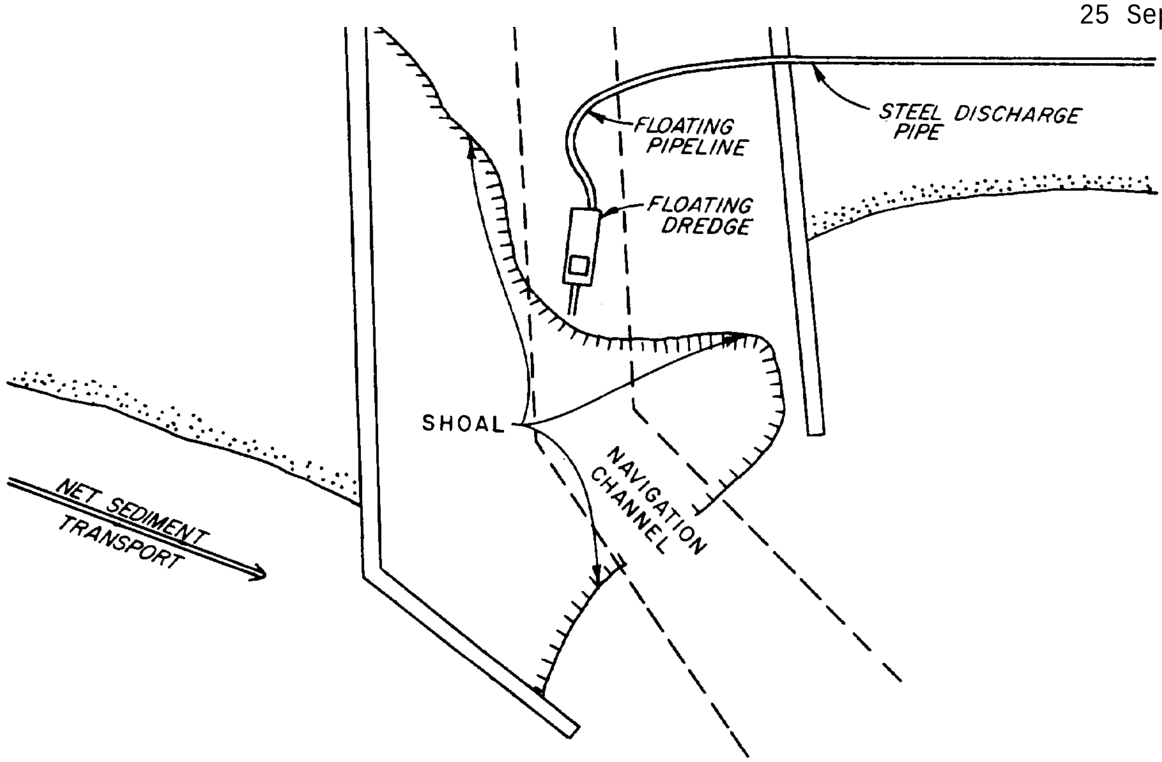


Figure 3-18. Bypass System, Santa Cruz, California.

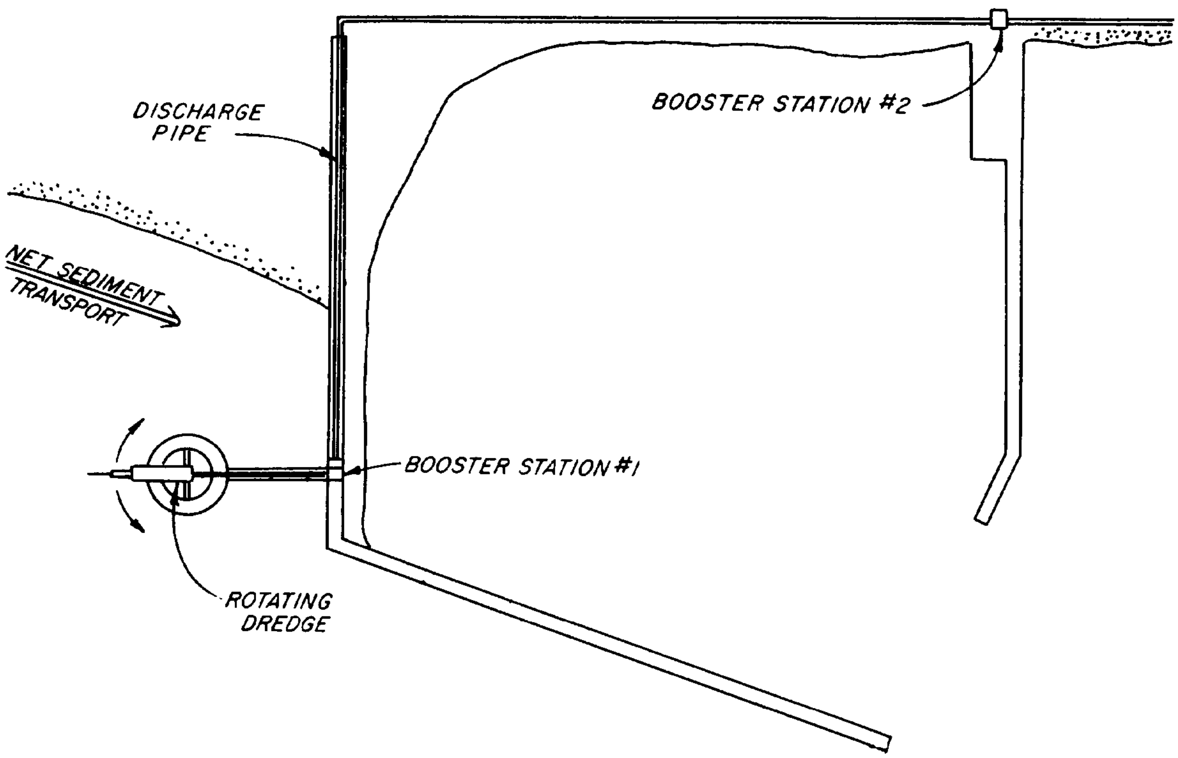


Figure 3-19. Bypass System, Marina di Carrara, Italy.

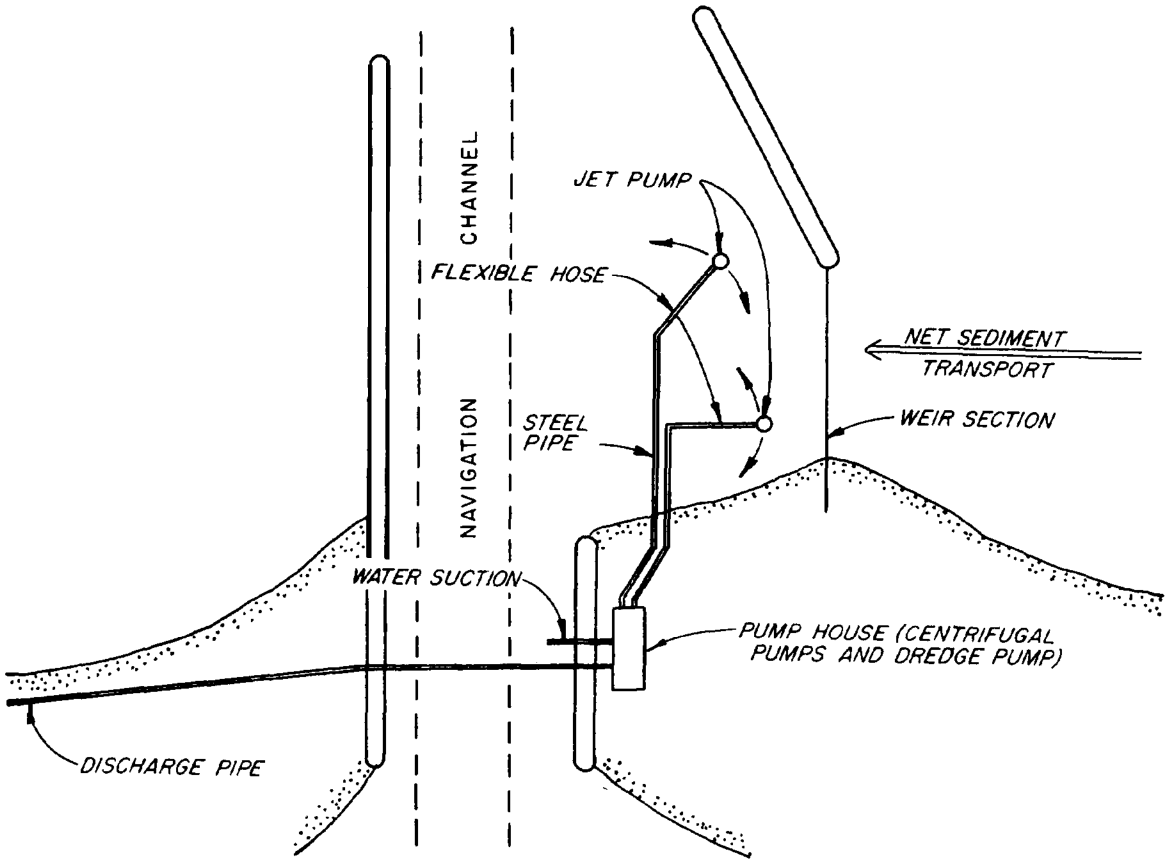


Figure 3-20. Bypass System, Rudee Inlet, Virginia.

beaches. The pump house contained centrifugal water pumps to drive the jet pumps and a dredge pump to boost sand through the discharge pipe. The system could cover most of the deposition area and remove sand at an average rate of 150 cubic yards per hour. Such a system could operate either periodically if the desposition area was cleaned initially by a hydraulic dredge or on a relatively continuous basis.

3-21. Environmental Impacts.

a. General. Environmental impacts generally fall into three categories: (1) dredging and disposal, (2) water quality impact of project during normal operation, and (3) induced erosion or accretion. Impacts of dredging and disposal are discussed in paragraph 3-18 and reference c.

b. Water Quality. Changes in the water circulation and basin flushing rate (water exchange) primarily impacts water quality in small-boat harbors. Changes in dissolved oxygen, temperature, nutrients, and toxic compounds also may be a problem. Water circulation and flushing rates usually can be predicted in physical models. If adverse water quality is predicted, the biological impact on affected organisms is needed. If impact is substantial, mitigation measures, with cost, must be developed. Implementation of a mitigation measure will depend on cost of mitigation, extent of impact, and the species affected. Flushing and circulation can be enhanced by rounding the corners of basins, sloping or stepping basins downward toward the entrance channel, designing for a length/width ratio close to one, and minimizing depth to the point of adequate navigability. Floating breakwaters may be desirable to mitigate water quality problems. Water exchange culverts from basins to adjacent water bodies should allow open channel flow because submerged culverts result in lower discharges than open channel culverts for the same head difference. Leaving a gap between the breakwater and the shore can improve water circulation and exchange rates, and reduce cost. This design also allows unblocked migration routes for some fish species.

c. Erosion and Accretion. Boat basin breakwaters and entrance channels can block littoral drift movement. The result is generally accretion behind the breakwater on the updrift side, possible channel shoaling, and downdrift erosion. Prediction of the erosion and accretion magnitude is needed and cost of suitable mitigation measures must be developed. Implementation of mitigation measures will depend on cost and value of property affected.

3-22. Physical Models.

a. General. As a general rule, physical model studies are needed for final design of small boat navigation projects. These model studies optimize the design and verify suitable project performance. Physical model investigations of small-craft harbors generally are conducted to do the following:

(1) Determine the most economical breakwater configurations that will provide adequate wave protection for small craft in the harbor.

(2) Quantify wave heights in the harbor.

(3) Alleviate undesirable wave and current conditions in the harbor entrance and provide harbor circulation.

(4) Provide qualitative information on the effects of structures on the littoral processes.

(5) Study flood and ice flow conditions.

(6) Study shoaling conditions at the harbor entrance.

(7) Study long-period oscillations in the harbor.

(8) Study tidal currents or seiche generated currents in the harbor.

(9) Stabilize inlet entrances.

(10) Develop remedial plans for alleviation of undesirable conditions as found necessary.

(11) Determine if modifications to existing projects could be made that would reduce construction cost significantly and still provide adequate harbor protection.

b. Scale Selection. During the planning and design phases of a physical model investigation of harbor problems, the model scale must be determined. Scale selection normally is based on the following factors:

(1) Depth of water required in the model to prevent excessive bottom friction effects.

(2) Absolute size of model waves.

(3) Available shelter dimensions and area required for model construction.

(4) Efficiency of model operation.

(5) Available wave-generating and wave-measuring equipment.

(6) Model construction costs.

Normally, geometrically undistorted models (i.e., both the vertical and horizontal scale are the same) are necessary to ensure accurate reproduction of short-period wave and current patterns (i.e., simultaneous reproduction of both wave refraction and wave diffraction).

c. Example of Design Optimization. The Port Ontario Harbor project is an excellent example of how physical models can optimize design. A three-dimensional harbor model (Figure 3-21), which tested 11 different layouts, was used to determine the best plan for economy, wave protection, and channel shoaling (Bottin 1977). Two-dimensional model tests of breakwater stability and overtopping (Figure 3-22) also were conducted. Three breakwater plans were tested which indicated that the crest width could be reduced from four-stone diameter to three-stone diameter without sacraficing stability (Carver and Markle 1981). This change resulted in a substantial cost savings. A layout of the recommended plan is shown in Figure 3-23.

3-23. Mathematical Models. Mathematical models are generally used to evaluate the following:

a. Basin layout (long-period wave penetration)

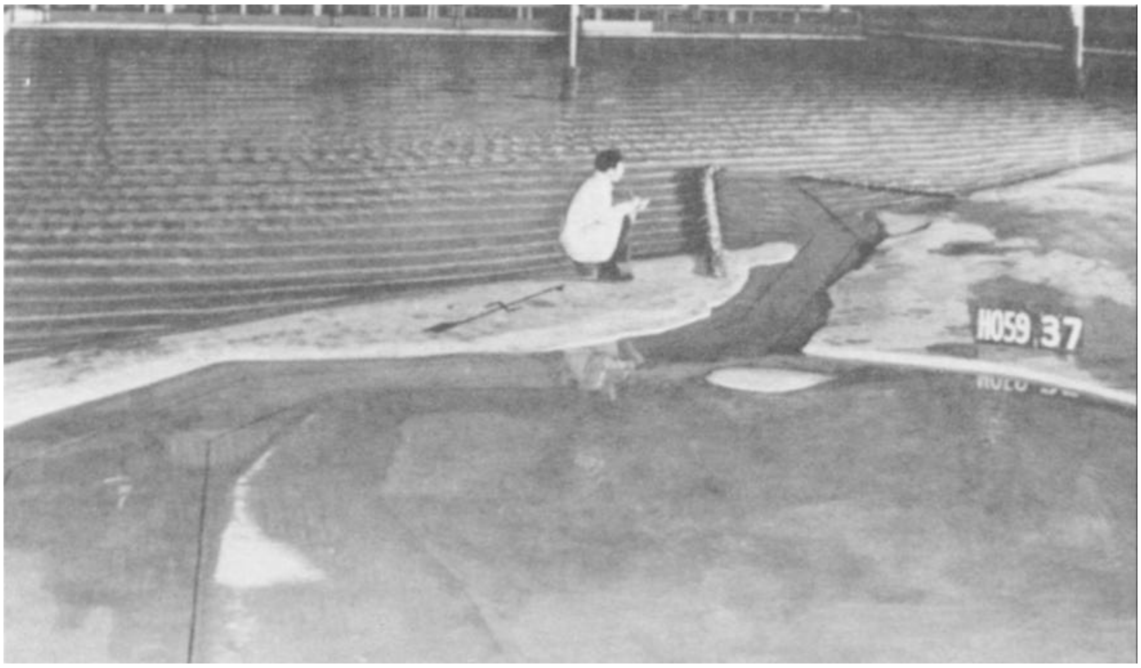


Figure 3-21. Three-dimensional model of Port Ontario Harbor, New York.


Figure 3-22. Two-dimensional model of Port Ontario Harbor breakwater.

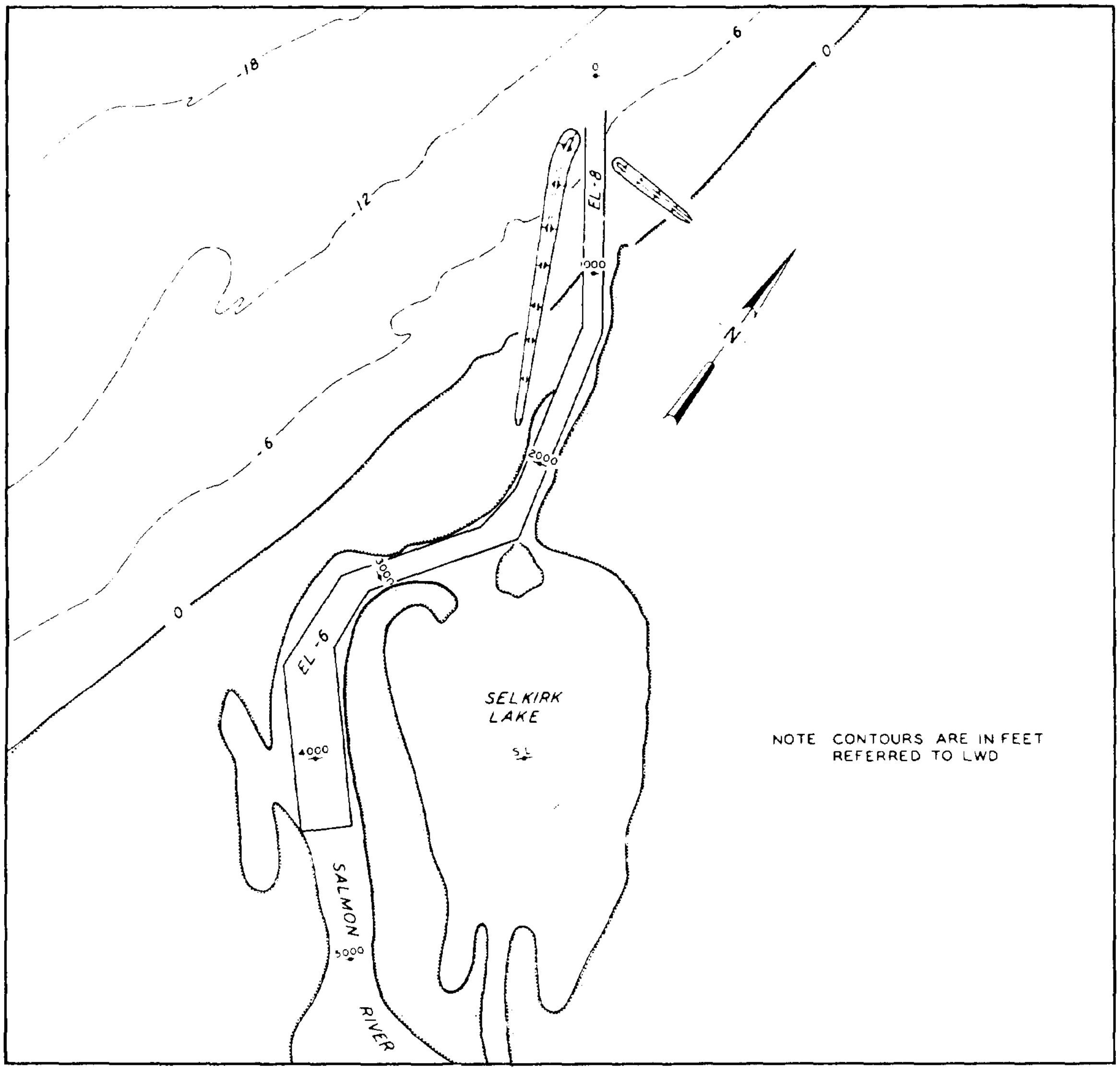


Figure 3-23. Recommended improvement plan, Port Ontario Harbor, New York.

b. Floating breakwater performance and mooring loads

c. Ship motion

d. Harbor oscillation

e. Tidal characteristics

f. Tsunamis effects, etc.

Mathematical models are often less expensive to conduct, provide quicker answers, are appropriate for preliminary design and screening alternatives, and allow examination of conditions within a framework of physical models.

3-24. Lessons Learned.

a. General. Various harbor sites studied are categorized into the following classifications:

(1) Open coast harbors built seaward/lakeward from the shoreline and protected by breakwaters.

(2) Harbors built inland with an entrance through the shoreline.

(3) Harbors built inside a river/stream mouth.

(4) Entrance/Inlet harbors.

Some advantages and disadvantages of each harbor classification considering both functional and economic aspects are discussed below. Also addressed are typical problems frequently encountered for each harbor classification along with some potential problems and/or considerations to be aware of.

b. Harbor Classes.

(1) Open Coast Harbors Built Seaward/Lakeward From the Shoreline and Protected by Breakwaters. Numerous harbors of this type are situated along the ocean coastlines and the Great Lakes (Figure 3-24). Some harbors are built along a straight shoreline and protected entirely by breakwaters while others are constructed in coves or irregularities in the shoreline. Harbors constructed seaward/lakeward from the shoreline generally require less dredging than harbors built through the shoreline since their entrances and basins are normally in deeper water. Due to the greater depths, however, more stone is usually required for construction of protective breakwaters. Generally, when breakwaters enclosing a harbor extend and terminate in relatively deep water, shoaling in the entrance channel is minimized and the requirement for

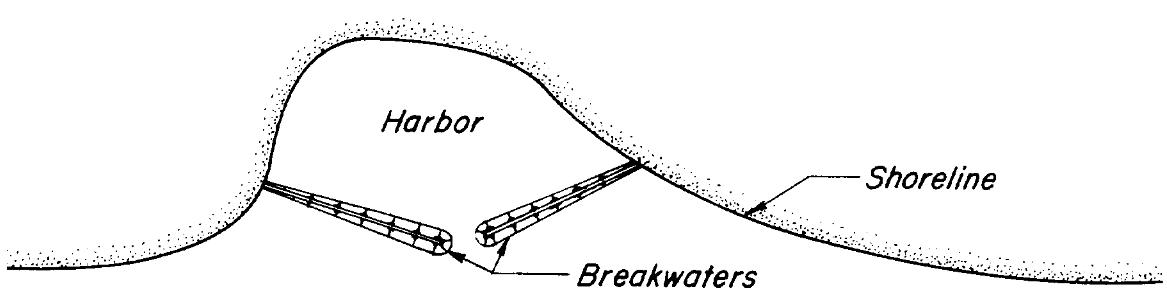


Figure 3-24. Example of a typical open coast harbor built seaward/lakeward from shoreline and protected by breakwaters.

maintenance dredging is reduced or eliminated. A study of the littoral processes should be conducted, however, since breakwaters extending into deep water may prevent natural bypassing and result in sediment accretion on the updrift side and erosion on the downdrift side if the net longshore transport rate is not zero. Harbors of this type often are built in coves or irregularities in the shoreline where natural land features aid in providing wave protection and reduction of breakwater lengths. In many cases, the construction of a single structure to provide protection for waves from the predominant direction of storm wave attack is satisfactory. Caution must be exercised before using a single strucutre, however, in that it could intercept the movement of sediment for less frequent waves from other directions and result in harbor shoaling.

(2) Harbors Built Inland With an Entrance Through the Shoreline. Many harbors of this type are located along the Great Lakes and ocean shorelines (Figure 3-25). In most instances, an existing lake, embayment, marsh area, etc., situated close to the shoreline is used as the harbor with the entrance being dredged from the shoreline to the embayment, lake, etc. Harbors constructed inland with entrances through the shoreline normally require more dredging than other harbor classes. In many instances, however, a channel may be dredged from the shoreline to the existing lake, embayment, lagoon, etc., and result in minimal dredging. Since the harbor is located inland, it is sheltered from storm wave activity, and normally only minimum breakwater lengths constructed in relatively shallow water are required to provide wave protection to the entrance. Common problems, however, with breakwaters terminating in shallow water (in the breaker zone) are (1) shoaling of the entrance, and (2) undesirable crosscurrents in the entrance both of which could be potentially hazardous to small-craft navigation. These factors must be addressed prior to harbor construction.

(3) Harbors Built Inside River/Stream Mouths. Numerous small-boat harbors are situated in river mouths along the shorelines of the Great Lakes and oceans (Figure 3-26). These harbors normally require a minimum of dredging. Small-boats are usually sheltered from large waves and, like harbors built inland, normally minimum breakwater lengths constructed in shallow water are required to provide wave protection to the entrance. Problems with entrance shoaling and undesirable cross-currents in the entrance caused by wave action or tidal currents may be experienced and shoaling due to sediment transported downstream may occur. The structures must be positioned so they do not interfere with the passage of flood and/or ice flows in the river/stream. The harbor also should be located inside the river/stream mouth so that it is protected from flood flows (high velocity river/stream currents) which may result in damage to small boats and/or harbor facilities.

(4) Entrance/Inlet Harbors. Numerous small-boat harbors are located in inlets along the ocean coasts (Figure 3-27). These harbors are normally protected from heavy wave action. Dredging requirements normally exist at the inlet opening, and usually only minimum breakwater lengths constructed in

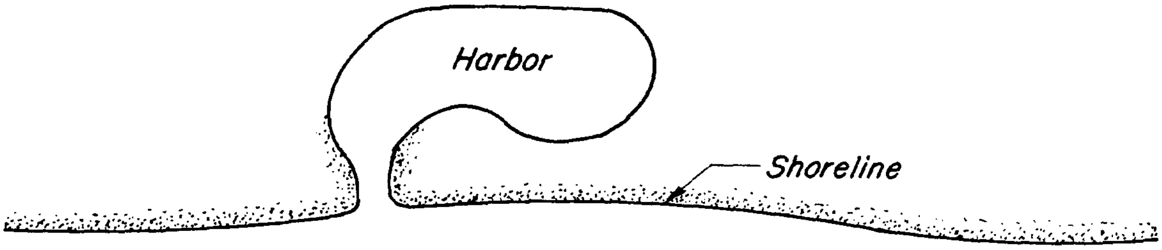


Figure 3-25. Example of a typical harbor built inland with entrance through shoreline.

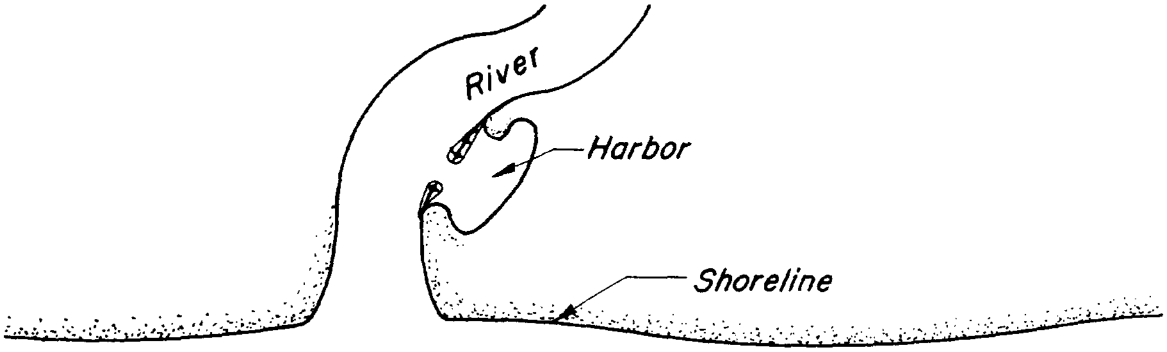


Figure 3-26. Example of a typical harbor built inside a river mouth.

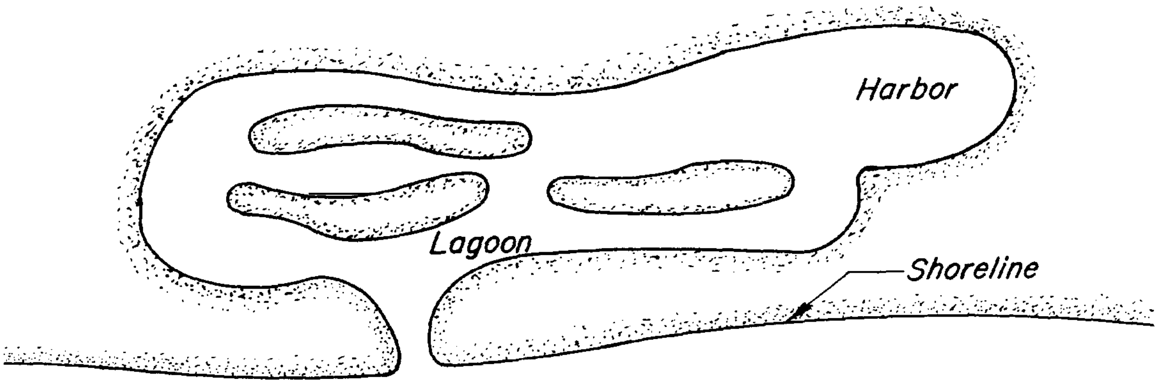


Figure 3-27. Example of a typical entrance/inlet hrbor located within a lagoon.

shallow water are required to provide wave protection to the entrance. In some cases, however, jetties must be long enough to extend beyond the ebb tidal deltas. Again shoaling problems and currents may be encountered in the entrance due to wave action and tides resulting in navigational difficulties. Stabilization of the inlet opening is a major concern for these studies. Tidal exchange between the ocean and embayment or lagoon may create high velocity flood and ebb currents through the entrance. Sediments moving alongshore are influenced by these currents and create a meandering unstable entrance. In some cases, weirs are installed in jetties in conjunction with dredged deposition basins. These systems are designed to intercept material moving alongshore and prevent sediments from moving into the inlet entrance where they may come under the influence of tidal currents. Deposition basins require periodic maintenance dredging to remain effective. Some sand bypassing schemes are discussed in paragraph 3-19.

3-25. Operation and Maintenance (O&M). A comprehensive plan of how the project will be operated and maintained is required. This plan is presented in support of the operation and maintenance costs. The following elements are normally included in the O&M plan.

a. Predicted Project Costs and Physical Changes. Include the post construction prediction of physical changes and anticipated O&M costs.

b. Surveillance Plan. Describe the type and frequency of post construction surveys. These could be hydrographic, beach profile, tide and wave records, and jetty stability. The plan covers minimum monitoring of project performance to verify safety and efficiency. Surveys may be needed to establish unacceptable project performance and the basis for corrective measures. Surveys will also be needed before and after periods of maintenance and repair.

c. Analysis of Survey Data. Comparative studies of the survey data are required. These comparative studies verify design information such as rates of erosion, shoaling, and jetty deterioration.

d. Periodic Inspections and Project Performance Assessment. Present a tentative periodic inspection schedule. Inspections include a site assessment. and a comparison of survey data to project changes predicted during the design effort. Compare actual project O&M costs to predicted cost.